CHAPTER 5

FOUNDATION ANALYSES

Section I. Bearing Capacity of Wall Foundations

5-1. Analysis Principles and Methods.

a. EM 1110-2-1903. A discussion of the principles and methods involved in analyzing bearing capacity is contained in EM 1110-2-1903. The manual concludes that Terzaghi's general bearing capacity equation, $q = CN_c + wz\, N_q + WbN_w$, is preferred. However, the manual does not address modifying the general equation for effects of embedment, inclined loads, sloping bases, passive-type wedges with sloping surfaces, overburden pressure, and eccentric loads (moment-induced stresses), all of which are needed for computing the bearing capacity of retaining and flood walls. The computer program CBEAR (Appendix O) can assist in these computations.

b. Mode of Failure. The mode of failure depends on the relative compressibility of the soil, loading conditions, and geometric considerations (Vesic 1975). This manual is restricted to general shear failure of shallow strip foundations, i.e., those whose widths are greater than their embedment. A general shear failure normally exists for dense sand and stiff clay. However, for loose sand and soft clay, which may occur more frequently for flood walls constructed in a flood plain, the bearing capacity should be computed based upon local shear conditions (Vesic 1975).

c. Factor of Safety. The FS is calculated as follows:

$$FS = \frac{Q}{N'} \qquad [5\text{-}1]$$

where

N' = effective normal force applied to the base of the structure

Q = normal component to the base of the structure of the ultimate bearing capacity

The minimum acceptable bearing capacity factors for retaining walls and inland and coastal flood walls are listed by loading case in Tables 4-1 through 4-3. For each loading case, the same loadings as determined by the overturning analysis should be used. Options to consider in the event of inadequate bearing capacity have been presented in paragraph 4-20.

5-2. General Bearing Capacity Equation.

The general bearing capacity equation for a strip footing is:

$$Q = \bar{B}\left[(\xi_{cd}\xi_{ci}\xi_{ct}\xi_{cg}cN_c) + (\xi_{qd}\xi_{qi}\xi_{qt}\xi_{qg}q_oN_q) + \frac{(\xi_{\gamma d}\xi_{\gamma i}\xi_{\gamma t}\xi_{\gamma g}\bar{B}\gamma N_\gamma)}{2}\right] \quad [5\text{-}2]$$

where

Q = normal component of the ultimate bearing capacity of the foundation

$\bar{B}$ = effective width of the base (B - 2e, as shown in Figure 5-1)

B = width of the geometric base (as shown in Figure 5-1)

e = eccentricity of the load with respect to geometric base width

c = cohesion parameter of the foundation

ξ = factors as explained in paragraphs 5-4 through 5-8

N_c, N_q, N_γ = bearing capacity factors for a strip load

q_o = effective overburden pressure on the plane passing through the base of the footing

γ = effective unit weight of the foundation material, $\gamma_{buoyant}$ below water table, γ_{moist} above

Figure 5-1 illustrates the meanings of all of the terms required to use the information given in paragraphs 5-3 through 5-8. The general bearing capacity equation is taken from the CBEAR user's guide (Mosher and Pace 1982) (see also Appendix O). The appropriate soil foundation shear strength for retaining walls and inland and coastal flood walls is listed, by loading case, in Tables 4-1 through 4-3.

5-3. Bearing Capacity Factors. Bearing capacity factors for a horizontal strip footing under vertical loading are:

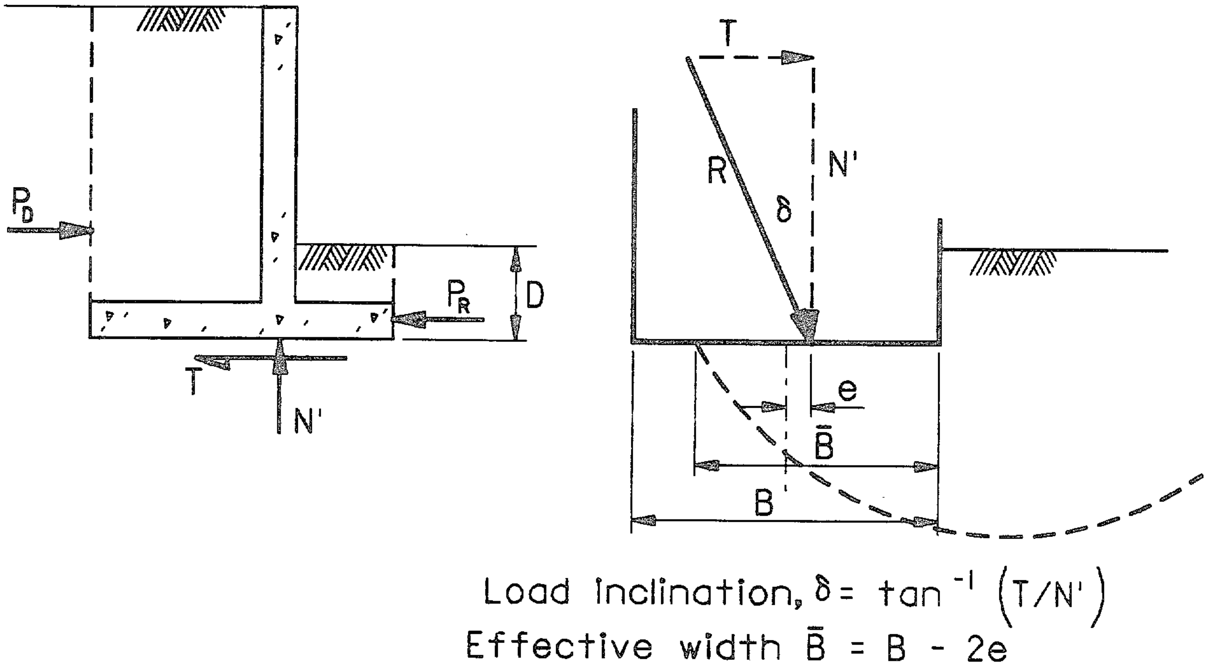
$$N_q = \left[e^{(\pi \tan \phi)}\right] \tan^2 \left(45° + \frac{\phi}{2}\right) \quad [5\text{-}3a]$$

$$N_c = (N_q - 1) \cot \phi \qquad (\text{when } \phi > 0) \quad [5\text{-}3b]$$

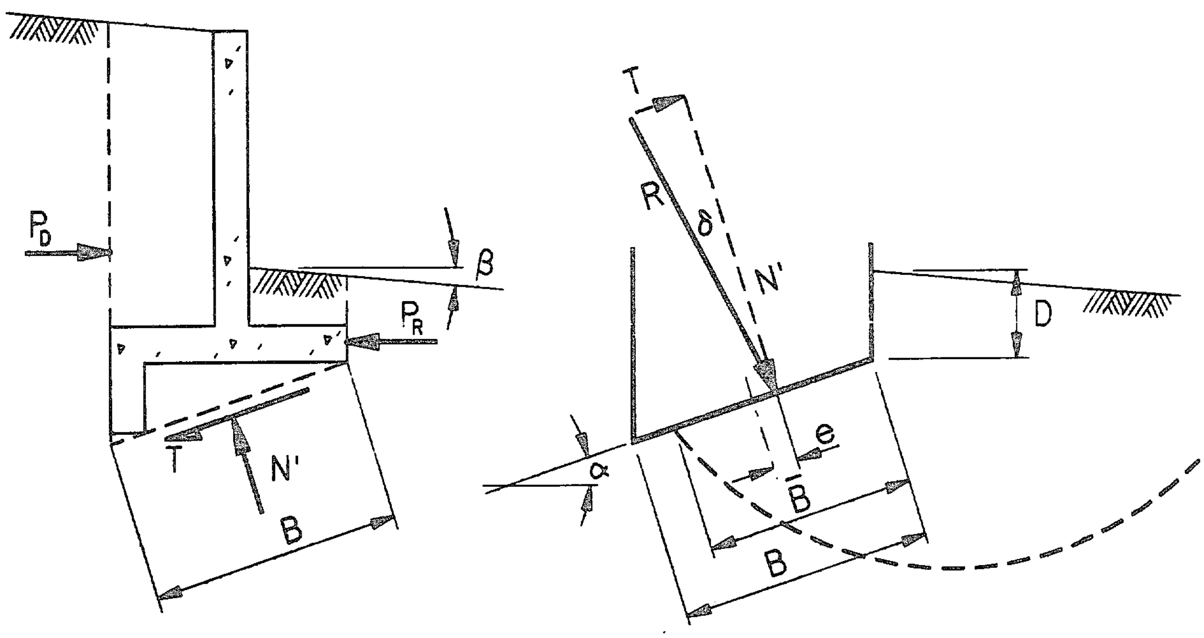
or

$$N_c = 5.14 \qquad (\text{when } \phi = 0) \quad [5\text{-}3c]$$

$$N_\gamma = (N_q - 1) \tan (1.4\phi) \quad [5\text{-}3d]$$



a. Horizontal base



b. Keyed base, sloping ground

Figure 5-1. Terms used in bearing capacity equation

Bearing capacity factor values for ϕ, ranging between 0 and 50 degrees, are given in Table 5-1.

5-4. Embedment Factors. Embedment factors take into consideration the shearing resistance along the foundation slip plane that exists in the soil above the base of the footing, on the toe side of a wall. These factors can be computed as:

$$\xi_{cd} = 1 + 0.2\left(\frac{D}{\bar{B}}\right) \tan\left(45° + \frac{\phi}{2}\right) \qquad [5\text{-}4a]$$

$$\xi_{qd} = \xi_{\gamma d} = 1 \qquad (\text{when } \phi = 0°) \qquad [5\text{-}4b]$$

or

$$\xi_{qd} = \xi_{\gamma d} = 1 + 0.1\left(\frac{D}{\bar{B}}\right) \tan\left(45° + \frac{\phi}{2}\right) \qquad (\text{when } \phi > 10°) \qquad [5\text{-}4c]$$

When ϕ lies between 0 and 10 degrees, a linear interpolation can be made for $\xi_{\gamma d}$ between 1 for $\phi = 0°$, and $1 + 0.1(D/B) \tan(45° + \phi/2)$ for $\phi = 10°$. Embedment factors account for the shear strength above the base of the footing. Their use may be unconservative if the shear strength does not exist.

5-5. Inclination Factors. Inclination factors account for the effect of load inclination for concentrically loaded foundations. They are computed as follows:

$$\xi_{qi} = \xi_{ci} = \left(1 - \frac{\delta°}{90°}\right)^2 \qquad [5\text{-}5a]$$

$$\xi_{\gamma i} = \left(1 - \frac{\delta°}{\phi}\right)^2 \qquad [5\text{-}5b]$$

Where δ is the angle that the line of action of the load makes with a line drawn normal to the base. If $\delta > \phi$, $\xi_{\gamma i}$ should be set equal to zero.

Table 5-1

Bearing Capacity Factors (CBEAR User's Guide)*

ϕ	N_c	N_q	N_γ	$\tan \phi$	$\tan^2\left(45° + \frac{\phi}{2}\right)$
0	5.14	1.00	0.00	0.0000	1.0000
1	5.38	1.09	0.00	0.0175	1.0355
2	5.63	1.20	0.01	0.0349	1.0723
3	5.90	1.31	0.02	0.0524	1.1105
4	6.19	1.43	0.04	0.0699	1.1500
5	6.49	1.57	0.07	0.0875	1.1910
6	6.81	1.72	0.11	0.1051	1.2335
7	7.16	1.88	0.15	0.1228	1.2776
8	7.53	2.06	0.21	0.1405	1.3233
9	7.92	2.25	0.28	0.1584	1.3709
10	8.34	2.47	0.37	0.1763	1.4203
11	8.80	2.71	0.47	0.1944	1.4716
12	9.28	2.97	0.60	0.2126	1.5250
13	9.81	3.26	0.74	0.2309	1.5805
14	10.37	3.59	0.92	0.2493	1.6382
15	10.98	3.94	1.13	0.2679	1.6984
16	11.63	4.34	1.37	0.2867	1.7610
17	12.34	4.77	1.66	0.3057	1.8263
18	13.10	5.26	2.00	0.3249	1.8944
19	13.93	5.80	2.40	0.3443	1.9655
20	14.83	6.40	2.87	0.3640	2.0396
21	15.82	7.07	3.42	0.3839	2.1171
22	16.88	7.82	4.07	0.4040	2.1980
23	18.05	8.66	4.82	0.4245	2.2826
24	19.32	9.60	5.72	0.4452	2.3712
25	20.72	10.66	6.77	0.4663	2.4639
26	22.25	11.85	8.00	0.4877	2.5611
27	23.94	13.20	9.46	0.5095	2.6629
28	25.80	14.72	11.19	0.5317	2.7698
29	27.86	16.44	13.24	0.5543	2.8821

(Continued)

* (Mosher and Pace 1982).

Table 5-1 (Concluded)

ϕ	N_c	N_q	N_γ	tan ϕ	$\tan^2\left(45° + \frac{\phi}{2}\right)$
30	30.14	18.40	15.67	0.5774	3.0000
31	32.67	20.63	18.56	0.6009	3.1240
32	35.49	23.18	22.02	0.6249	3.2546
33	38.64	26.09	26.17	0.6494	3.3921
34	42.16	29.44	31.15	0.6745	3.5371
35	46.12	33.30	37.15	0.7002	3.6902
36	50.59	37.75	44.43	0.7265	3.8518
37	55.63	42.92	53.27	0.7536	4.0228
38	61.35	48.93	64.08	0.7813	4.2037
39	67.87	55.96	77.33	0.8098	4.3955
40	75.31	64.20	93.69	0.8391	4.5989
41	83.86	73.90	113.99	0.8693	4.8149
42	93.71	85.38	139.32	0.9004	5.0447
43	105.11	99.02	171.15	0.9325	5.2893
44	118.37	115.31	211.41	0.9657	5.5500
45	133.88	134.88	262.75	1.0000	5.8284
46	152.10	158.51	328.74	1.0355	6.1260
47	173.64	187.21	414.34	1.0724	6.4447
48	199.26	222.31	526.47	1.1106	6.7865
49	229.93	265.51	674.94	1.1504	7.1536
50	266.89	319.07	873.88	1.1918	7.5486

5-6. Base Tilt Factors. These factors are used to take into account the effect of a sloping base. The base tilt factors are computed as:

$$\xi_{qt} = \xi_{\gamma t} = (1 - \alpha \tan \phi)^2 \quad (\alpha \text{ in radians}) \qquad [5\text{-}6a]$$

$$\xi_{ct} = 1 - \left(\frac{2\alpha}{\pi + 2}\right) \quad (\alpha \text{ in radians}) \quad (\text{when } \phi = 0°) \qquad [5\text{-}6b]$$

$$\xi_{ct} = \xi_{qt} - \frac{1 - \xi_{qt}}{N_c \tan \phi} \quad (\text{when } \phi > 0°) \qquad [5\text{-}6c]$$

where α is the angle the slip plane of the structural wedge makes with the horizontal, measured in radians. The sign of α will follow the sign convention given in Chapter 4.

5-7. Ground Slope Factors. Ground slope factors are used to correct for a sloping ground surface on the toe side of the wall. The factors are computed as:

$$\xi_{\gamma g} = \xi_{qg} = [1 - \tan (\beta)]^2 \qquad [5\text{-}7a]$$

$$\xi_{cg} = 1 - \left[\frac{2\beta}{(\pi + 2)}\right] \quad (\beta \text{ in radians}) \quad (\text{when } \phi = 0°) \qquad [5\text{-}7b]$$

$$N_\gamma = -2 \sin \beta \quad (\text{when } \phi = 0°) \qquad [5\text{-}7c]$$

$$\xi_{cg} = \xi_{qg} - \frac{1 - \xi_{qg}}{N_c \tan \phi} \quad (\text{when } \phi > 0°) \qquad [5\text{-}7d]$$

where β is the angle the ground surface makes with the horizontal, measured in radians. β is positive when the ground slopes down and away from the footing.

5-8. Effective Overburden Pressure. q_o is defined as the effective vertical stress due to the soil and/or surface loads above the base of the footing, on the toe side of the wall, as follows:

$$q_o = \gamma' D \qquad [5\text{-}8a]$$

where

γ' = effective unit weight of the overlying soil

D = depth from the soil surface to the base of the structural wedge

For the special case of a sloping surface, compute q_o as:

$$q_o = \gamma' D \cos [ABS(\beta)] \qquad [5\text{-}8b]$$

5-9. Combination of Factors. As discussed in the CBEAR user's guide (Mosher and Pace 1982), the correction factors for the load inclination, base tilt, and ground slope and the adjustment for the load eccentricity should only be used in unison when all of these factors tend to produce failure in the same direction.

5-10. Example. Example problems using the general bearing capacity equation are presented in Appendix N.

Section II. Other Considerations

5-11. Settlement.

a. EM 1110-2-1904. A discussion on the various factors involved in the settlement of a structure, on methods for estimating settlements, and on the limitations in the accuracy of conducting settlement analyses from laboratory tests is contained in EM 1110-2-1904. The principles and methods presented are applicable to a majority of civil works projects. Additional information for unique or special projects can be obtained from various texts on soil mechanics. The computer program CSETT (Appendix O) can assist in performing a settlement analysis.

b. Allowable Settlement. The maximum value of angular distortion (settlement/length of structure) which can be tolerated without cracking of reinforced concrete retaining walls is 0.002 to 0.003 radian (Duncan and Buchignani 1976).

5-12. Deep-Seated Sliding. A deep-seated sliding analysis should be performed to check for sliding within weak layers which may exist beneath structures. The analysis should be in accordance with procedures outlined in paragraph 4-16. Active and passive wedges should be located a sufficient distance apart to allow a rotational slip surface to develop. Generally, a slip plane inscribed in an arc with a radius equal to the height of the active wedge will comply with this requirement (Figures 5-2 and 5-3). When the wall is resting on thick strata of weak soils, shallow shear failure should be

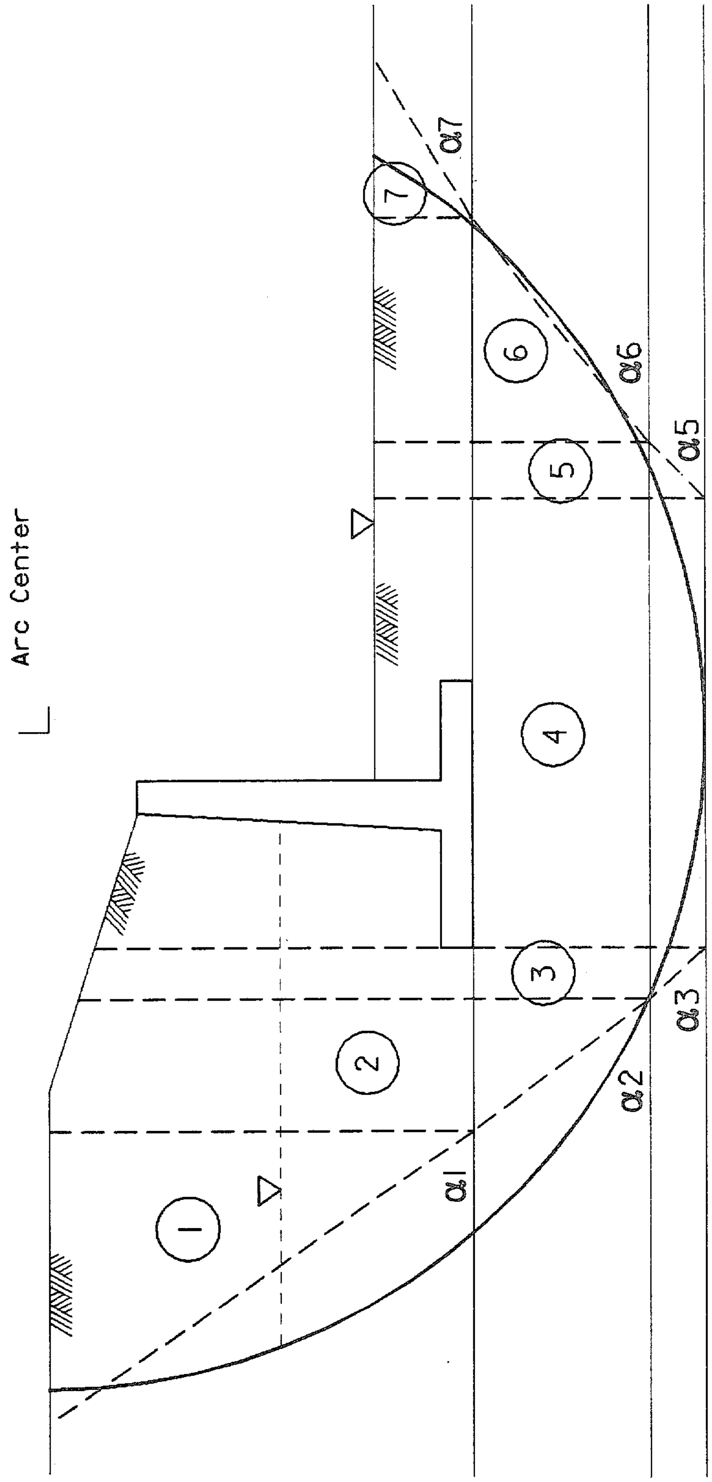


Figure 5-2. Deep-seated sliding analysis with vertical face of driving wedge at heel

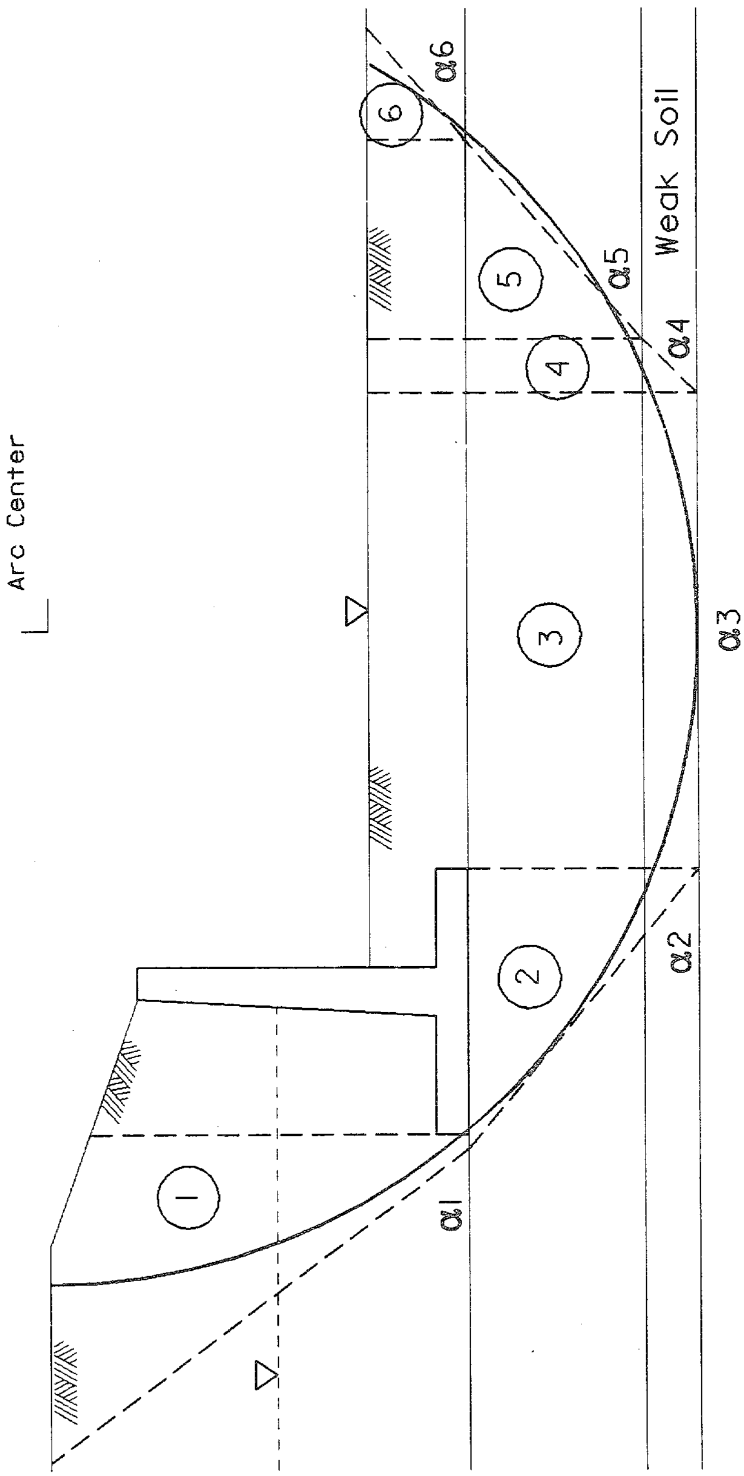


Figure 5-3. Deep-seated sliding analysis with vertical face of driving wedge at toe

investigated. This may be assumed to occur below the base of the retaining or flood wall along a cylindrical surface passing through the heel (Figure 5-4). The minimum factor of safety, which must not be less than 1.5, is determined by trial and error by changing the center of the trial circle.

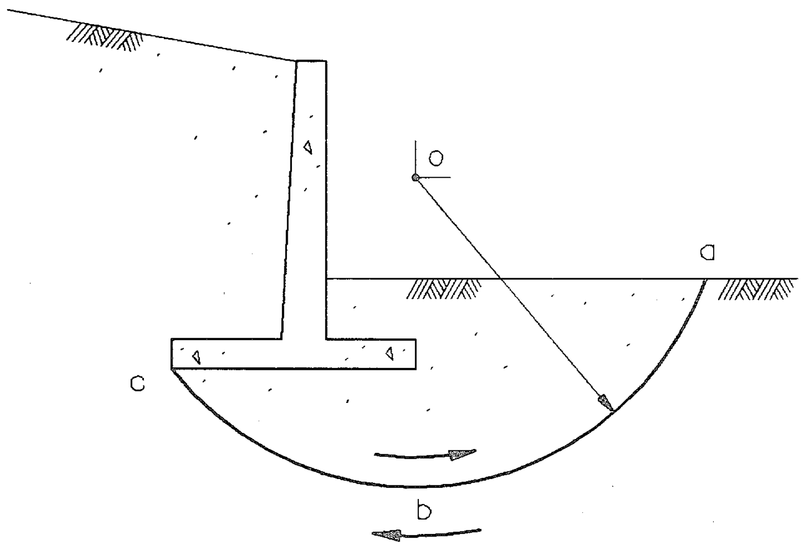


Figure 5-4. Shallow shear surface

5-13. Liquefaction Susceptibility. Where walls are underlain by sands below the water table in seismically active areas, an analysis should be made of the safety against foundation liquefaction. Flood walls in alluvial valleys are particularly likely to be situated over loose, saturated sands that may be liquefiable. A preliminary assessment of liquefaction susceptibility can be made using Seed's simplified method (Seed 1976, Seed and Idriss 1982) which is based on the standard penetration test. If the foundation is found to be non-liquefiable, no further analysis need be made. If liquefaction may occur, an assessment should be made of the risks and consequences of liquefaction failure and the benefits and costs of alleviating the risks. The occurrence of an earthquake during a flood is a case of the joint occurrence of independent rare events. For flood walls, the probability (risk) of an earthquake during a flood will be much smaller than the probability during a non-flood period, but the associated consequences may be much higher. For certain walls, (e.g., a low retaining wall remote from other structures) the probability of liquefaction failure and the related consequences may translate into such a small risk that accepting the risk may be the preferred alternative. Possible alternatives to dealing with potentially liquefiable foundations include:

a. Changing the proposed location (usually the best alternative, where feasible).

b. Removing and replacing the liquefiable materials.

c. Improving the liquefiable materials in place, by densification or grouting.

d. Accepting the risks and consequences of liquefaction.